Earth Surface Processes and Landforms
Earth Surf. Process. Landforms **24**, 665–675 (1999)

RESISTANCE TO SHALLOW LANDSLIDE FAILURE THROUGH ROOT-DERIVED COHESION IN EAST COAST HILL COUNTRY SOILS, NORTH ISLAND, NEW ZEALAND

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Received 5 December 1997; Revised 6 November 1998; Accepted 17 November 1998

ABSTRACT

Measured shear strength (cohesion and friction) of airfall-derived east coast hill country regolith is insufficient for the maintenance of existing regolith depth/slope angle relationships in the catchment of Lake Waikopiro in northern Hawke's Bay, New Zealand. If these regoliths are attributed with an additional increment of cohesion derived mechanically from a turf mat membrane, existing depth/slope angle distributions are explicable. Sites where failure has occurred have been measured, and back-analysis used to derive a range of values for additional mechanical cohesion. Values are in the range 0·2–7·6 kPa, consistent with earlier findings. This range is narrowed further, to 0·7–6·9 kPa, with assumptions regarding soil moisture status at failure. Copyright © 1999 John Wiley & Sons, Ltd.

KEY WORDS: earthflow; factor of safety; mechanical cohesion; critical slope angle; turf mat

INTRODUCTION

Over the past century, substantial changes have occurred in the New Zealand hill country. Specifically, there has been extensive deforestation, scrub and fern clearance and conversion to pastoral land uses. Erosion has become widespread following this removal of indigenous vegetation, greatly exceeding already high natural levels (Page and Trustrum, 1997). Hill country regolith is in the process of adjusting to a new distribution of depth and slope angle in equilibrium with the mass and energy regime under these contemporary land uses (DeRose *et al.*, 1991). Rainfall-initiated shallow regolith landslides have become the most significant erosional process, producing the greatest amounts of sediment for redistribution (Page *et al.*, 1994). Failure typically occurs at the interface between the relatively unweathered bedrock and a regolith that is dominated by airfall deposits. Using the terminology of Cruden and Varnes (1996), these failures can be termed earth slides or, more appropriately given the liquid behaviour they exhibit, earthflows.

The inherent shear strength of the regolith is generally considered to be the principal factor in defining limiting conditions for slope failure. Accordingly, critical values of slope and depth have been calculated using strength parameters derived from direct shear tests. However, field observations clearly show that calculated critical values do not adequately represent the field situation. There are numerous sites where the depth and angle of stable regolith units exceed calculated critical values. The regolith must therefore derive resistance to failure from a source other than its inherent strength, as determined through conventional testing procedures. Field observation of the slope failure process may give some indication of other factors effectively controlling slope resistance. Pertinent to this is Crozier's (1996) description of incipient earthflows – features that appear to be characteristic of this terrain (Figure 1). These incipient earthflows are defined as features where a regolith unit has experienced internal deformation and possibly fluidization, but

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CCC 0197-9337/99/080665–11 $17.50

Figure 1. An incipient earthflow, Mangaweka, North Island, New Zealand, showing displacement under an unruptured membrane of pasture vegetation, but an absence of surface translation

lateral translation has not occurred. The absence of translational movement characteristic of this phenomenon is attributed to the constraining influence of a surface membrane of densely interwoven roots of pasture species.

This paper describes the strength behaviour of regolith in the catchment of Lake Waikopiro in northern Hawke's Bay, an area that is representative of the erosion-prone hill country on the east coast of New Zealand's North Island. It is proposed that the inherent strength of these soils is insufficient to maintain observed regolith depths and slope angles, and that additional mechanically derived cohesion is required to sustain these regoliths.

SOILS OF THE WAIKOPIRO CATCHMENT

Soils of the study area are formed on a variety of Tertiary marine sediments, from mudstones to sandstones. However, in many localities æolian materials – tephra and tephric loesses – constitute the dominant component of the regolith. The region, lying downwind of the Central Volcanic Zone, contains some 14 identified Holocene tephras (Eden *et al.*, 1993). The Waiohau Ash (11 250 years BP $\pm$ 200) is the basal tephra of soils of the region (Pullar, 1973). Both Taupo and Waimihia tephras (1850 and 3280 years BP, respectively) occur as pumiceous lapilli within upper (A and AB) horizons, although biological and colluvial processes have disrupted original bedding (Eden *et al.*, 1993). On much of the steep hillslopes, however, these tephras have been eroded, remaining only on the relatively undisturbed plateaux and interfluves.

The regional soil is the Taupo-Whakatane Yellow-Brown Pumice Soil (DSIR, 1963). Pohlen (1971) describes the moderately to strongly leached Patoka-Tutira Soils, which occur at Waikopiro, as intergrades between these Yellow-Brown Pumice Soils and Yellow-Brown Loams. They are formed from tephra overlying sandstones and mudstones. They are young, very friable and weakly weathered, containing the clay mineral allophane, and are characterized by sandy textures.

Typical soil profiles are exposed in erosional features. Most failure scars have undisturbed regolith in their exposed flanks and scarps, although a few have occurred in colluvium. In either case, regolith profiles are characterized by three distinct components. At the top is a dark, friable, low density soil that is interwoven with a dense network of pasture grass roots. This horizon typically extends down some 0·2–0·3 m,

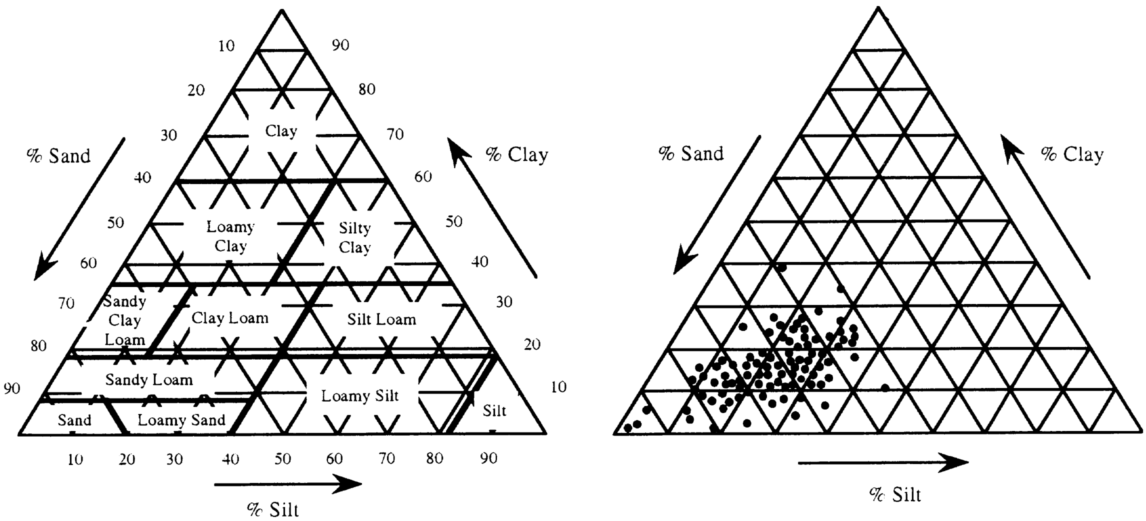

Figure 2. (a) Soil texture classes (Milne *et al.*, 1991); (b) Textures of Waikopiro soils ($n = 125$)

constituting approximately 15 per cent of the exposed profile. Below it is a horizon that in undisturbed regoliths consists of minimally weathered airfall material, while in colluvial regolith this horizon is darker (suggesting the incorporation of organic soils) and contains angular clasts up to several centimetres in diameter. These horizons are largely homogeneous for physical properties and effectively constitute the remainder of exposed profiles. At the base of the regolith is a thin band of material that is similar in many physical respects to the overlying regolith (e.g. particle size distribution, plasticity), but is noticeably less dense and proves weaker under penetrometer testing. This band is thought to represent the location of the failure plane. It separates the airfall and colluvial regoliths from underlying Tertiary bedrock, and may have been weakened by eluviation caused by lateral waterflow immediately above the relatively impermeable bedrock surface.

Analysis shows that clay contents of Waikopiro soils range from 1·0 to 39·0 per cent, averaging 15·37 per cent ($\pm$ 1·65 per cent; 95 per cent C.I.). Some 67 per cent of soil samples can be defined as Sands, Sandy Loams or Loamy Sands (definitions follow Milne *et al.*, 1991; see Figure 2), i.e. they can be considered cohesionless, or likely to possess little cohesion. One-third of the samples are Clay Loams with one sample of each of Loamy Clays and Loamy Silts. The low clay content of these soils has implications for strength, as both cohesional and frictional components of strength are very much influenced by the presence of clay particles. The development of cohesion is enhanced by the large surface area provided by clay minerals, while particle size distribution has a large influence on interlocking friction – a measure of the amount of deviation required for particles at the shear surface to move past each other. Clearly, larger particles will require greater deviation. Conversely, a soil dominated by platy clay particles will require very little such deviation. Skempton (1985) demonstrated that clay minerals have little effect on strength when they represent less than 20–25 per cent of the fine earth fraction, in which case the soil behaves as a silt/sand.

SHEAR STRENGTH OF WAIKOPIRO SOILS

In addition to their low clay content, Waikopiro soils can be considered cohesionless because of their stress history. On slopes where regolith has been subject to colluvial and landslide activity, these processes are thought to cause sufficient disruption and particle reorientation to remove any pre-existing cohesion, reducing strength to the residual state. Slow creep processes in otherwise stable soils are thought to have a

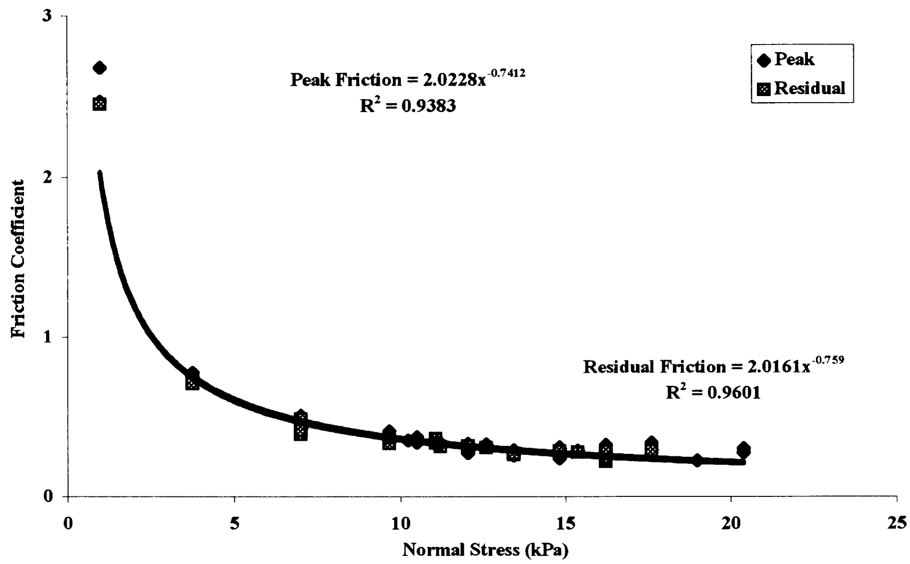

Figure 3. Relationship between normal stress and frictional strength. Note the minimal difference between peak and residual strengths (peak $n = 71$; residual $n = 31$)

similar effect. These assumptions are supported by the results of shear tests which show little difference in the strength measured on initial shearing (peak) and that mobilized along the already sheared surface (residual). Consequently, there are strong grounds for regarding the strength of Waikopiro soils as purely frictional.

Conventionally, peak and residual shear strengths obtained from a number of shear tests are plotted against normal stress, and a linear relationship derived. This relationship takes the form of Coulomb's equation:

$$s = c' + \sigma \tan \phi' \tag{1}$$

where s = shear strength, c' = cohesion (effective), σ = normal stress, ϕ' = angle of shearing resistance (effective). In reality, this linear relationship has been shown to be only an approximation, valid over a narrow range of normal stresses, and especially inappropriate at the low normal stresses characteristic of shallow soils (Kenney, 1984). At low normal stresses, such as those applying to shallow shear surfaces, volume change and dilation within the shearing soil has an influence, and frictional strength does not vary as a linear function of normal stress. As normal stress increases, and there is greater resistance to dilation under shearing, the ratio of frictional strength to normal stress decreases. Friction becomes an approximate constant in the relationship of strength to normal stress only where normal stress is greater than 150–200 kPa (Skempton and Petley, 1967; Townsend and Gilbert, 1973).

Shear surfaces within the Waikopiro regolith are shallow and not subject to large normal stresses. Even the deepest, densest, saturated regoliths on the gentlest slopes are only subject to normal stresses ranging from 20 to 60 kPa, while typical values are in the range 5–20 kPa. Frictional strength in these soils, therefore, varies according to normal stress. Assuming cohesion is absent, friction can be isolated from the normal stress component of strength by normalizing values of strength derived from testing, i.e. dividing strength by normal stress. The resultant is the friction coefficient (ϕ') in tanform.

A series of laboratory direct shear tests was performed on samples taken from typical shear plane locations. At least three shears were performed for each sample (in most cases, considerably more) over a range of normal stresses appropriate to the site from which the sample was taken (i.e. slope angles, regolith depths and weighted material densities are known for each of these sites, enabling calculation of *in situ* normal stresses).

All shear test results were then plotted together to derive a relationship between normal stress and frictional shearing resistance (Figure 3). The relationship takes the form of a power function, with the frictional component of strength decreasing to a negative power of normal stress:

$$\tan\phi' = 2\cdot 0161\sigma^{-0\cdot 759} \tag{2}$$

where ϕ' = effective angle of shearing resistance (degree) and σ = normal stress (kPa). This relationship is generally consistent with what is known of friction, in the sense that its significance decreases with increasing normal stress, and in that it approaches a constant value. However, it is clear that the decrease in the significance of friction is very rapid, and that an approximately constant value is achieved at considerably lower values of normal stress than those suggested in the literature, where friction has been found to approach a constant value at normal stresses on the order of 150–200 kPa. Current data show friction approaching a constant value with normal stresses an order of magnitude lower than this, and are thus inconsistent with earlier observations. This apparent anomaly is explicable, however, if the range of normal stresses at which friction is expected to become constant is considered. It would seem reasonable to expect that a constant value of friction should be approximately equivalent to a value characteristic of the material's particle size distribution. The question then becomes: at which range of normal stresses should we expect to find a frictional coefficient consistent with particle size distribution? Values reported in the literature are specific to clay shales, i.e. materials with characteristic frictional coefficients that are an order of magnitude lower than those expected for silty sands. For example, Townsend and Gilbert (1973) found constant frictional values ranging from 3·1° to 10·8°, which is generally appropriate to the particle size distribution of clay shales. This occurs with normal stresses of approximately 150 kPa. Present results indicate an approximately constant value of friction that is consistent with an expected value specific to silty sands, i.e. in the range 27–35°. This occurs, however, at a much lower range of normal stress: 4–6 kPa. A possible interpretation of the present data, therefore, is that the range of normal stresses at which constant frictional coefficients are encountered is itself a dynamic variable and is specific to the material in question.

CRITICAL SLOPE ANGLES

Application of strength values derived from Equation 2 to slopes in the Waikopiro basin clearly indicates that the resistance of the slope/regolith unit itself must be greater than the measured strength of shear zone soils alone. This is demonstrated by calculation of theoretical critical slope angles using laboratory-derived strength and comparing these with the range of slope angles of undisturbed regolith actually occurring in the field. In terms of factor of safety analysis, critical conditions are those defining the threshold between stability and instability. Following Henkel and Skempton (1954), the factor of safety for an infinite slope is defined as the ratio of shear strength to shear stresses:

$$FS = \frac{c' + (\gamma - m\gamma_w)z\cos^2\beta\tan\phi'}{\gamma z\sin\beta\cos\beta} \tag{3}$$

where c' = effective cohesion (kPa), γ = regolith density (kg m^{-3},) γ_w = density of water (981 kg m^{-3}), m = ratio of water table height to regolith thickness, z = regolith thickness (m), β = angle of shear plane (degree) and ϕ' = effective angle of shearing resistance (degree).

Rearrangement of Equation 3 allows derivation of critical slope angles (β_c):

$$\tan\beta_c = \frac{\gamma - m\gamma_w}{\gamma}\tan\phi' \tag{4}$$

Equation 4 has been used to calculate critical slope angles for 15 failure sites for which values of the above variables have been measured (Table I). The only variable for which values are not known is the height of the water table at the time of failure. Accordingly, critical slope angles have been modelled for a range of

Table I. Observed depths, slope angles and densities at 15 failure sites

Site	Av. depth (m)	Slope (deg.)	Dry density (kg m^{-3})	Saturated density (kg m^{-3})
A	1·00	30	1242	1744
B	1·05	22	1201	1695
C	1·05	35	1333	1798
D	1·61	30	1209	1695
E	1·13	27	1355	1787
F	1·37	29	934	1489
G	0·72	27	1158	1646
H	1·02	36	1213	1706
I	1·15	33	891	1494
J	0·86	34	1217	1707
K	1·39	36	1008	1595
L	1·32	21	1471	1827
M	1·62	22	1597	1894
N	1·16	20	1378	1845
O	0·91	29	1357	1812

Table II. Observed slope angles at 15 failure sites, and critical slope angles calculated for a range of m values

Site	Slope (deg.)	Critical Slope (deg.) for $m =$				
		0·00	0·25	0·50	0·75	1·00
A	30	20·3	15·8	12·2	9·4	7·1
B	22	18·3	14·0	10·7	8·2	6·1
C	35	20·2	16·0	12·6	9·9	7·6
D	30	14·8	11·3	8·6	6·6	4·9
E	27	16·9	13·4	10·6	8·2	6·3
F	29	19·6	13·9	9·9	7·0	4·9
G	27	25·7	19·8	15·1	11·4	8·5
H	36	22·4	17·3	13·4	10·2	7·7
I	33	24·2	17·0	12·0	8·5	6·0
J	34	24·2	18·9	14·6	11·2	8·4
K	36	20·6	14·9	10·9	8·0	5·8
L	21	13·3	10·8	8·6	6·8	5·3
M	22	10·8	9·0	7·3	5·9	4·6
N	20	15·1	12·0	9·5	7·4	5·8
O	29	20·1	16·0	12·7	9·9	7·7

porewater pressure values. Density was calculated as a function of saturated (γ_s) and dry (γ_d) densities and of water table height (m):

$$\gamma = (\gamma_s m) + [\gamma_d(1 - m)] \tag{5}$$

Dry and saturated densities are composite values for the entire soil column, weighted according to the proportion that each horizon contributes to the whole. Angles of internal friction were derived using Equation 2, with normal stress calculated as a function of slope angle, average regolith depth and the modelled density:

$$\sigma = \gamma z \cos^2 \beta \tag{6}$$

Results are presented in Table II. Assuming $m = 0{\cdot}5$, the maximum critical slope angle is not much more than 15°, and even with no offset of normal load by positive porewater pressure (i.e. $m = 0{\cdot}0$) the maximum possible stable slope is 25·7°. Yet slopes consisting of stable regolith of similar depth exist at steeper angles than this, immediately adjacent to the landslide scar sites from which these critical angles were calculated.

Figure 4. A dense network of finely interwoven roots of pasture grass forming a 20–30 cm membrane at the top of an undisturbed airfall-derived soil profile

COHESIVE STRENGTH

Clearly there appears to be another mechanism through which these regoliths derive their resistance to failure. While shear tests confirm the absence of any significant soil cohesion, these tests were performed only on materials taken from the vicinity of the shear plane, i.e. at depths of approximately one metre. Strength parameters derived are therefore specific to the soils of this zone, while stability depends on the resistance generated by the slope/regolith unit in its entirety. Shallow interlocking root networks (Figure 4) can contribute mechanical reinforcement to soils (Sidle *et al*., 1985; Selby, 1993). For species similar to pasture in the Waikopiro study area, Selby (1993) gives values of additional apparent cohesion ranging from 0·1 to 9·8 kPa, and Lawrance *et al.* (1996) found that soils with grass root networks were in some cases stronger by between 0·8 and 11·6 kPa.

A mechanism such as surface membrane restraint derived from root cohesion may therefore account for the presence at Waikopiro of stable regolith at slope angles greater than those permitted purely by the frictional strength of soils in the shear zone. If values of friction derived as a function of normal stress are assumed for sites that have failed, a value for this increment of mechanical cohesion can be obtained through back-analysis. Assuming that at failure the ratio between inherent strength and shear stress was unity, Equation 3 can be rearranged to derive a value of cohesion at failure:

$$c' = (\gamma z \cos\beta \sin\beta) - [(\gamma - (m\gamma_w))z\cos^2\beta\tan\phi'] \tag{7}$$

For the reason given above, it was necessary to back-analyse using a range of porewater pressures. Cohesion was modelled for a range of porewater pressures, corresponding to water table to regolith depth ratios (m) of between 0·0 and 1·0, producing values of cohesion in the range 0·2–7·6 kPa (Table III). These derived cohesion values are consistent with the ranges given by Selby (1993) and Lawrance *et al.* (1996). Root-derived cohesion must generally be lower than the maximum values given in Table III, otherwise these regoliths would not be subject to failure. Investigations in the Makahu catchment of Taranaki's hill country, which is subject to similar landslide activity, indicate that failure may occur with porewater pressures developed with a water table height to regolith thickness ratio of $m = 0{\cdot}25$ (Crozier and Preston, 1998). If this ratio also applies in the Hawke's Bay region, then root-derived cohesion may lie in the range 0·4–5·4 kPa.

Table III. Values of cohesion (kPa) for 15 failure sites, calculated for a range of m values

Site	Cohesion (kPa) for m =					
	0·00	0·25	0·50	0·60	0·75	1·00
A	1·926	2·388	2·856	3·045	3·329	3·805
B	0·799	1·164	1·535	1·685	1·911	2·291
C	3·117	3·620	4·128	4·332	4·639	5·153
D	4·582	5·339	6·102	6·409	6·871	7·643
E	2·499	2·922	3·350	3·522	3·781	4·214
F	1·939	2·627	3·327	3·610	4·036	4·752
G	0·193	0·471	0·754	0·869	1·042	1·333
H	2·556	3·076	3·602	3·813	4·132	4·666
I	1·442	2·111	2·792	3·068	3·484	4·184
J	1·623	2·037	2·456	2·625	2·880	3·307
K	3·215	4·069	4·934	5·282	5·807	6·688
L	2·510	2·846	3·185	3·321	3·526	3·868
M	4·728	5·099	5·472	5·621	5·845	6·221
N	1·322	1·681	2·045	2·192	2·413	2·785
O	1·772	2·143	2·518	2·669	2·897	3·278

An alternative explanation that has been used elsewhere to account for the discrepancy between measured strength and actual slope resistance identifies the importance of soil water tension in providing an increment of strength in unsaturated soils (Brooks and Anderson, 1995). It has been suggested that the matric suction provided by negative porewater pressures may act as a stabilizing factor within the regolith. If suction were a critical component of additional strength, failure could be expected to occur whenever field capacity was exceeded. Evidence from an adjacent catchment indicates that this is not the case in the region involved in this study. Stable regolith has been observed in this terrain under saturated conditions (Merz, 1997), and tensiometer observations in potential landslide sites reveal the existence of perched water tables, even in relatively dry (low rainfall) conditions, with no indication of failure (E. Jensen, pers. comm., 1997).

Regolith saturation is promoted by both a shallow regolith overlying an extremely impermeable bedrock surface, and the incidence of very high and intense rainfalls, often associated with tropical cyclones. Glade (1997), modelling antecedent soil water conditions in this region using a water balance approach, has calculated that conditions of positive porewater pressure are common. More importantly, while all recorded failures have occurred under conditions of positive porewater pressure, these conditions do not always result in regolith failure, as would be the case if matric suction were the sole additional increment of resistance to failure. While positive porewater pressures therefore appear to be an important trigger for slope failure in these regoliths, there is no indication that failure is due to reduction of matric suction.

Rainfall intensity values and the recorded recurrence interval of landsliding can be used to further isolate an appropriate value of root-derived cohesion for a typical undisturbed regolith in this region. With an average depth of one metre and an average porosity of 0·48, a rainfall of 48 mm in one hour will saturate this regolith, assuming it is completely dry at the commencement of the rainfall event, and that drainage is minimal. One-hour rainfall intensities of this magnitude have return periods of between 20 and 50 years (Table IV). Landsliding occurs in this region, however, with a much greater frequency than this. As an example, six episodes of widespread landsliding were reported in the study area over the 23-year period, 1965 to 1988, giving a return period of approximately four years. The immediate implication of this is that failure is possible without complete regolith saturation. A one-hour rainfall intensity with a four-year return period is approximately 30 mm (Table IV). This is approximately 60 per cent of the value required for complete saturation and suggests that at failure the water table height to regolith depth ratio may be around $m = 0{\cdot}6$. Back-analysis on the 15 landslide scars to derive a value of cohesion, assuming $m = 0{\cdot}6$, produces a range of values for root cohesion of 0·9–6·4 kPa (Table III). In reality, however, as Glade (1997) has demonstrated, these regoliths commonly contain antecedent moisture, which would mean that m at failure will generally be higher than 0·6. As a first approximation, however, the range 0·7–6·9 kPa, being the minimum and maximum

Table IV. Predicted maximum rainfalls (mm) for selected durations and recurrence intervals (Thompson, 1987)

Return period (years)	Duration (h)						
	0·5	1	6	12	24	48	72
2	19	28	80	126	179	225	248
10	27	39	108	176	253	312	345
20	31	44	119	197	285	350	387
50	35	51	135	226	327	400	443

values with $m = 0{\cdot}5$ and $m = 0{\cdot}75$ respectively, is considered to represent minimum values for root-derived cohesion in this area.

SUMMARY AND DISCUSSION

The young airfall-derived soils that are widespread in the hill country of the east coast of New Zealand's North Island are essentially cohesionless. Their inherent strength, as measured in the potential shear zone, is almost exclusively frictional. The lack of cohesion in these soils can be attributed to their particle size distribution, their youth, and to the action of continual slope processes. The nearshore marine sedimentary rocks and tephras and tephric loesses that are the parent materials of these soils are composed largely of particles in the silt to fine sand size range. Further, these are young parent materials and have not yet been extensively weathered. Thus the soils they produce are relatively coarse grained, with low clay contents. The majority of these hillslope soils occur under conditions of continual shear stress, resulting in a range of slope processes. As a result, they have not developed any significant cohesional strength.

Frictional strengths have been determined through drained shear tests in the laboratory using a direct shear box. The low range of normal stresses occurring within the Waikopiro regolith precludes the use of the linear Coulomb strength relationship to derive constant values of friction. Instead, this has been approximated as a friction coefficient, i.e. strength is treated as purely frictional, decreasing to a negative power of normal stress. It appears, however, that slope resistance does not depend solely on the frictional strength of the regolith itself, as critical slopes derived from theoretical calculation are significantly lower than slope angles that exist within the catchment.

Although matric suction has been demonstrated to be an important component of slope resistance in some localities, observations of slope hydrology rule this out as a critical factor in the stability of the area under investigation. Continued stability of regolith slopes is attributed instead to the action of mechanical cohesion derived from the root networks of pasture grasses. In many instances mechanical cohesion provided by a turf mat membrane represents a significant component of slope resistance. Back-analysis has been used to determine a range of possible values for this increment of mechanical cohesion at sites where failure has occurred. These values are consistent with those reported in the literature for root strength cohesion in similar situations. A narrower range of values can be derived on the basis of assumptions regarding soil moisture status at failure.

It has been demonstrated that the exclusion of this increment of root resistance leads to unrealistic predictions of critical slope angles. Conventional testing of shear zone material fails to incorporate vegetation-derived resistance. Observations indicate that the pasture root mat can remain intact and act as a restraint to translational movement, even when failure has occurred within the underlying regolith. Incipient earthflows commonly exhibit some deformation at the surface, as flow lobes induce folding of the turf mat (Figure 5). While elevated porewater pressures may be capable of inducing internal deformation and fluidization within regolith units, downslope translation of landslide material takes place only when the turf membrane ruptures. Because in most cases this involves root pullout rather than rupture, measurement of root tensile strength may not properly represent the available resistance. This, and the deformation of the turf mat

Figure 5. The toe of an incipient earthflow, southern Hawke's Bay, with flow lobes under a folded but unruptured turf mat

itself that generally precedes rupture, present novel challenges to accurate replication, testing and measurement of this mechanism.

ACKNOWLEDGEMENTS

We would like to express our appreciation to the Guthrie-Smith Trust Board for their provision of accommodation in the Outdoor Education Centre at Tutira, and in particular to Paul Jennens and Matthew Cant of the GSOEC, for their co-operation, often at short notice and inconvenient times. Thanks also to Steve Reiri of Tutira Station and to the MacMillan family, on whose land we worked. The Physical Geography Board of Studies of Victoria University made substantial contribution to transport costs. We wish to acknowledge the assistance of the following people, who assisted with data collection on various occasions: Rhonda Catchpole, Mark Cormack, Frank Drost, Catherine Lambert, Yolanda Morgan, Maik Unbenannt and Penny Willis. Louisa Wolley helped in the determination of clay contents with a Rapid Sediment Analyser. Finally, thanks to Thomas Glade and an anonymous reviewer for helpful and thought-provoking comments on the manuscript.

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